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PREDICTING LANE UTILIZATION AT SIGNALIZED INTERSECTIONS IN ADVANCE OF ARTERIAL LANE DROPS

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RESEARCH

UDOT Keeping Utah Moving
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16. Abstract

When a through travel lane on an arterial road is dropped shortly beyond a signalized intersection, there can be a negative impact on capacity as drivers try to preemptively merge before the intersection. This research seeks to increase understanding of the factors that impact the utilization of discontinued (short) lanes and develop standards for design of lane drops to improve lane utilization. A total of 25 study locations throughout Utah were selected for analysis, and video recordings of the approaches were used to determine lane-by-lane counts during the PM peak period. Using the volume counts, the utilization rate of the short lane was calculated for each cycle and 15-minute period. A statistical analysis was used to predict the lane utilization rate based on several variables. Length until lane drop was significant with a positive influence on lane utilization while the natural log of through volume (in vehicles per hour), the presence of a trap lane, and speed limit were all significant with a negative influence on lane utilization. Use of the selected regression model should be used with understanding of the limitations of the dataset, specifically that the data was skewed toward lane drops with short lanes of 300 feet or shorter and approaches with less than 1,000 through vehicles per hour. Additionally, while the findings do not definitively indicate a preferable short lane length for acceptable lane utilization, it is apparent that it exceeds the typical lengths represented throughout Utah.

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LIST OF ACRONYMS

ATL	Auxiliary Through Lane
ATSPM	Automated Traffic Signal Performance Measures
NCHRP	National Cooperative Highway Research Program
UDOT	Utah Department of Transportation

EXECUTIVE SUMMARY

There are numerous locations throughout Utah where a through travel lane on an arterial road is dropped shortly beyond a signalized intersection. In such cases, often the discontinued (short) lane has a lower utilization rate than the other lanes of the approach as drivers try to preemptively merge before the intersection, which can negatively impact capacity. This research seeks to increase understanding of the factors that impact the utilization of the short lanes and develop standards for design of lane drops to improve lane utilization at the signalized intersection.

A total of 25 study locations throughout Utah were selected for analysis. The speed limit, number of approaching lanes, lane drop distance from the signalized intersection, and type of lane drop were noted for each location. Vehicles approaching each study location were recorded from 4 PM to 6 PM, and the timestamps and lane of each through vehicle entering the signalized intersection in advance of the lane drop location were collected. Gaps between each approaching vehicle were calculated and used to determine when cycles start and stop, a key step before performing data aggregations. Two aggregations were used: a cycle-level aggregation which assigned all vehicles to the cycle in which they occurred and a 15-minute aggregation which assigned all vehicles to the 15-minute bin in which their cycle began. For each cycle or 15-minute period in these aggregations, the lane utilization for the short lane was calculated as the volume in that lane divided by the average volume per lane.

An observational analysis revealed trends in the data between input variables and lane utilization. These trends, consistent for both aggregation types, included the following: a curved relationship between approaching through volume and lane utilization, an increase in lane utilization for an increase in the length of the short lane, a decrease in lane utilization for an increase in speed limit, and an increase in lane utilization for locations without a trap lane or a shared through/right lane. Knowledge of these trends assisted in selecting variables for the statistical model.

Ordinary least squares regression models were estimated through a stepwise process to estimate lane utilization based on several variables. The final model included four variables:

length of the short lane (measured as the distance from the signalized intersection to the end of lane striping, in feet), the natural log of approaching through vehicles per hour, the presence of a trap lane, and the speed limit. The length of the short lane was significant with a positive coefficient while the natural log of volume, the presence of a trap lane, and speed limit were all significant with a negative coefficient. It should be noted that most of the observations had short lanes of relatively short lengths (300 feet or shorter) with relatively low volumes (900 vph or less) and were associated with lane utilizations less than 0.6. This indicates that the predictive power of the model is skewed towards undesirable lane drop scenarios and that if there is a preferable short lane length for acceptable lane utilizations it likely exceeds the lengths represented in the sample.

It is recommended that the selected regression model be used with engineering judgment to predict lane utilization for lane drop locations in Utah less than 400 feet long and on approaches with less than 1,000 through vehicles per hour. It is also recommended that lane drop locations throughout Utah with longer short lanes and higher volumes be added to this study. Such an addition would allow for the fitting of a regression model to a more robust dataset and provide an opportunity for finding lane drop configurations associated with higher lane utilization.

1.0 INTRODUCTION

1.1 Problem Statement

There are numerous locations throughout Utah where a through travel lane on an arterial road is dropped shortly beyond a signalized intersection due to a lane reduction, a trap lane, or the end of an auxiliary lane. This discontinuation of a through lane requires drivers to decide where and when to merge from the short (discontinued) lane to the adjacent through lane. Observations of these situations have shown that many drivers will merge upstream of the signalized intersection. This behavior can result in non-ideal lane utilization at the traffic signal and unused roadway capacity. Some traffic analysis tools (such as Synchro) do not account for this underutilization by default and may lead roadway designers and decision makers to believe that the traffic benefits of additional intersection approach lanes will be greater than they ultimately are. Other software packages (such as VISSIM) can account for this preemptive merging behavior but require the traffic engineer to decide in their analysis when vehicles will begin merging. This decision can be difficult due to the lack of data on which to base it. The purpose of this research is to provide the Utah Department of Transportation (UDOT) with the data needed to help roadway designers and decision makers be more informed about the effects of lane drops downstream of signalized intersections and the true capacity benefits associated with roadway widening projects.

For the purposes of this research, locations immediately downstream of a signalized intersection where the roadway narrows by one through lane are considered lane drop locations. This includes lane reductions (cases where a through lane merges into an adjacent through lane), trap lanes (cases where a through lane is dropped because it becomes a turn lane at a downstream intersection or driveway), and auxiliary lanes (cases where an additional lane was added a short distance upstream of the signalized intersection, only to be dropped a short distance downstream of the intersection). All lane drop locations studied in this research narrowed by the right-most through lane.

1.2 Objectives

The primary objective of this research project is to determine the effects the distance from a signalized intersection to a lane drop location has on lane utilization upstream of the intersection and to provide guidance on a minimum short lane length. To this end, data pertaining to the number of vehicles using each lane at signalized intersections upstream of lane drop locations were collected, analyzed, and compiled into graphs and tables. This information will provide traffic engineers with tools to better estimate traffic performance based on actual driver behavior, giving roadway designers and decision makers more reliable information regarding the trade-offs associated with lane drop configurations and the effect they will have on the utilization of roadway capacity.

1.3 Scope

The scope of this research includes performing a literature review, collecting data from video recordings of the selected study locations, analyzing that data, and documenting the research findings. Videos were recorded for two hours at 25 locations with two- or three-lane approaches and downstream lane drops. The traffic volumes were aggregated and statistically analyzed to determine the correlation of lane utilization to traffic volumes and the length of the short lane. Graphs and tables were developed to illustrate the resulting relationships. The scope of this research project does not include any safety analysis or the development of software analysis guidelines or mitigation measures.

1.4 Outline of Report

The body of this report is organized in the following manner:

- Chapter 1 introduces the research topic, objectives, scope, and report outline.
- Chapter 2 provides a literature review on relevant topics.
- Chapter 3 describes the data used in the research and the methods used to collect it.
- Chapter 4 discusses the analyses performed on the data, including the development of the statistical model and a presentation of the findings.
- Chapter 5 provides a summary of the research, its findings, and its limitations.
- Chapter 6 presents the recommendations resulting from this study.

2.0 LITERATURE REVIEW

2.1 Overview

This chapter summarizes and reviews the literature for lane utilization at signalized intersections preceding lane drop locations. The search for relevant research was conducted primarily using the Transport Research International Documentation database of the Transportation Research Board. Both journal articles and reports at the state and national levels were included in the review. The first section in this chapter will discuss the literature pertaining to auxiliary through lanes (ATLs), and the second section will discuss the literature on forced merges and conversions to turn lanes.

2.2 Auxiliary Through Lanes

ATLs are lanes that are added immediately upstream of an intersection and are removed immediately downstream of an intersection. Such lanes are typical solutions for increasing intersection capacity, but they can negatively impact upstream lane utilization (NASEM 2011). Tainter et al. (2018) used driving simulations to predict lane utilization at ATLs on an individual level. They suggest that signage can play a key role in an individual's decision of lane usage that could mitigate suboptimal lane utilizations. Bugg et al. (2012) studied intersection-related variables that impact lane utilization at intersections with an ATL and a continuous through lane. They collected the following data at 8 intersections for 12 hours each: queue lengths in each lane, time to clear the intersection, arrivals on green, the average green time to cycle-length ratio, vehicle type, and lane utilization. The authors indicate that, in addition to through volume and the ratio of green time to cycle length, utility of an ATL is a function of each driver's arrival time and the queue lengths at the time of arrival. The National Cooperative Highway Research Program (NCHRP) report 707 (NASEM 2011) was commissioned to provide guidelines for and assess impacts of ATLs. The project included an analysis of 22 intersection approaches with ATLs including intersections with either one or two continuous through lanes and intersections with either dedicated right-turn lanes or shared ATL right-turn lanes. The authors created a tool to predict utilization for ATLs and shared ATL right-turn lanes based on effective green time, cycle length, volume (both through and right), saturation flow rate (both through and right), and intersection width. McCoy and Tobin (1982) found that auxiliary lane use is positively correlated with auxiliary lane length and negatively correlated with green time, and that right-turn volume does not impact lane choice for through vehicles if it is less than 25% of the through volume.

Ring and Sadek (2012) looked at both intersections with ATLs and intersections with lane drops. They collected data during the AM peak, PM peak, and lunchtime peak for 7 intersections (including 3 ATLs and 4 lane drops), including volumes for each lane and movement, heavy vehicle volume, and distance merging vehicles took to merge. They estimate a regression equation predicting utilization based on through volume, right-turning volume, median two-way-left-turn lane presence, upstream right-side trip-generation density, and downstream right-side trip generation.

2.3 Forced Merges and Conversions to Turn Lanes

Lee et al. (2005a and 2005b) studied intersections immediately upstream of arterial lane drop locations, either from forced merges or conversion to a turn-only lane, or where multiple left-turn lanes fed into a roadway that dropped one of those lanes. They studied 94 sites in North Carolina for 3 hours each. The primary variables for prediction were the length of the short lane and traffic volume. Additional data collected were taper lengths (or distance to first pavement indication of a required turn), storage lengths for turning vehicles, distance to next intersection, merge-related signs and locations, pavement markings, driveway number and activity level, presence of two-way-left-turn lanes, left-turn lane length (when applicable), speed limits, land use, lane width, grade, volume per cycle, saturation headway, queue lengths and delay, splits for each phase (green, yellow, all-red, and red) for each cycle, heavy vehicle percentage, and lane utilization. They separately estimated regression equations (considering transformations as well) for each of six different intersection types: two through lanes to one through lane with exclusive right-turn lane, two through lanes to one through lane with shared through and right-turn lane, two left-turn lanes to one lane at intersections, two left-turn lanes to one lane onto ramps, three through lanes to two through lanes with exclusive right-turn lane, and three through lanes to two through lanes with shared through and right-turn lanes. Variables that were significant in at least one of the models were short lane length, average volume per lane, number of merge-related signs, taper length, right-turn volume (for shared lanes) and heavy vehicle percentages. This project was very influential for design standards in North Carolina in the years since the publishing of the final report. One of the authors, now employed at North Carolina Department of Transportation, indicated that the results from the project are still regularly used today in design standards for lane drops, with the department using a general rule of thumb of keeping short lane distances above a quarter mile (Hummer 2020).

2.4 Summary

These relevant studies provide a framework for data collection and expectations for this study. The differing scopes of these studies make it clear that the classification and differentiation of intersection and lane drop type is critical to adequately predict lane utilization. These fundamental differences in types of intersections and lane drops outline the importance of collecting ample quality data for each scenario desired. The most thorough of all resources reviewed included data from 94 different sites with 12 data points (15-minute periods) for each site (Lee et al., 2005a) and was able to estimate regression equations for six different scenarios.

The literature that was reviewed highlighted a number of different variables for collection and analysis. The most important variables were length of short lane, volume by lane, and right-turning volume. The literature also revealed several variables that could also be important in predicting lane utilization in some cases. These variables were driveway information, saturation flow rate, ratio of green time to cycle length, taper length, presence of two-way-left-turn lane / left-turn storage length, heavy vehicle percentages, merge-related signage, speed limits, land use, distance to next intersection, intersection and lane widths, and queue lengths.

3.0 DATA COLLECTION

3.1 Overview

The scope of this research includes the study of 25 locations in Utah. This chapter discusses the data collection process. First, there will be a discussion on location selection. Following that is a description of the roadway data collection process. Then, there will be an explanation of the vehicle data collection.

3.2 Location Selection

For ease of data collection, only study locations along the Wasatch Front were considered. Locations with large driveways between the signalized intersection and the lane drop location were not selected in an effort to avoid having cases where the presence of a driveway influences the upstream lane utilization. Similarly, trap lane locations were not selected if the turn lane was expected to have significant volumes. Engineers on the research team relied on manual review and their familiarity with the locations to judge if any location should not be considered due to a high-volume driveway or turn lane. The distance from the upstream signal to the lane drop location was also considered, as it was desired to have a variety of lengths present in the dataset.

A total of 25 study locations in Utah were selected. These locations are listed in Table 3-1 alongside their lane count upstream of the signalized intersection, speed limit, and length from signalized intersection to lane drop location measured by the short lane length and taper length. A discussion on how the roadway data (speed limit and lengths) were obtained is given in the following section.

Table 3-1 Selected Locations

ID	*Description*	*Direction*	*UDOT Region*	*City*	*Lanes*	*Speed Limit (mph)*	*Short Lane Length (ft)*	*Taper Length (ft)*	*Total Length (ft)*
1	12th St & Wall Ave	NB	1	Ogden	3	40	170	610	780
2	12th St & Wall Ave	SB	1	Ogden	3	40	230	270	230
3	12th St & Wall Ave	EB	1	Ogden	3	40	70	430	500
4	12th St & Wall Ave	WB	1	Ogden	3	40	80	100	100
5	5600 S & 3500 W	NB	1	Roy	2	45	170	270	170
6	5600 S & 3500 W	SB	1	Roy	2	45	280	160	360
7	5600 S & 3500 W	WB	1	Roy	2	40	920	220	920
8	U.S. 89 & Skyline Dr	SB	1	South Ogden	3	55	90	180	360
9	S.R. 39 & S.R. 126	NB	1	West Haven	2	50	770	540	770
10	S.R. 36 & Saddleback Blvd	SB	2	Tooele County	3	60	2,100	unknown	2,100
11	1000 N & Redwood Rd	NB	2	Salt Lake City	2	45	470	290	470
12	1300 S & State St	EB	2	Salt Lake City	2	30	90	290	90
13	2300 E & Foothill Dr	EB	2	Salt Lake City	3	40	240	360	240
14	2100 S & 1300 E	NB	2	Salt Lake City	2	35	170	120	170
15	S.R. 111 & 7800 S	NB	2	West Jordan	2	50	370	450	370
16	8000 S & State St	SB	2	Midvale	3	40	100	280	100
17	9000 S & Redwood Rd	EB	2	West Jordan	3	40	260	170	260
18	9000 S & Redwood Rd	WB	2	West Jordan	3	40	280	160	280
19	9000 S & 700 W	WB	2	Sandy	3	40	180	400	180
20	9000 S & State St	NB	2	Sandy	3	40	200	110	200
21	S.R. 92 & 4800 W	EB	3	Highland	2	45	690	350	700
22	Main St & State St	EB	3	American Fork	3	35	140	210	140
23	1600 N & State St	WB	3	Orem	2	40	330	260	350
24	Center St & 1200 W	EB	3	Orem	3	35	100	320	300
25	University Pkwy & Geneva Rd	SB	3	Orem	2	45	260	230	260

Note: NB = Northbound, EB = Eastbound, SB = Southbound, and WB = Westbound.

3.3 Roadway Data Collection

Two of the lane drop characteristics needed for this research were approach speed limit and distance from the upstream intersection. These two characteristics were helpful in the data analysis process as will be described in further detail in Chapter 4.

The speed limit for each study location was collected using the UDOT Speed Limit KML file available on the UDOT Digital Delivery website (UDOT 2021). For the locations that were not a state route, the speed limit was determined from posted speed limit signs as found in the street view feature of Google Maps (Google 2021a). As shown in Figure 3-1, the approaching speed limits vary between 30 and 60 mph with nearly half of the approaches having a speed limit of 40 mph.

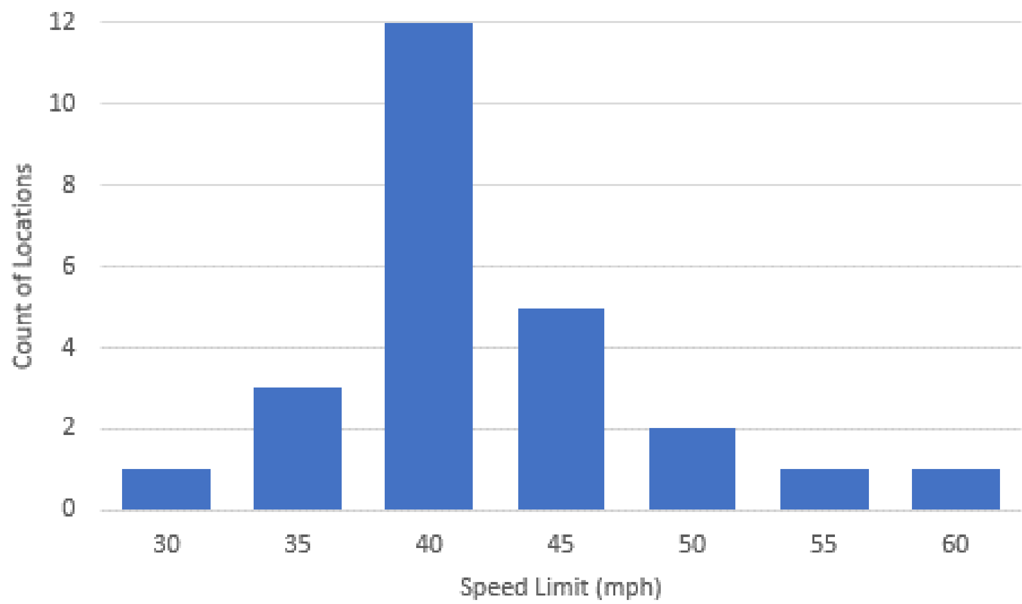


Figure 3-1 Distribution of speed limits.

As shown in Figure 3-2, the short lane length was measured as the distance between the downstream edge of the intersection and the end of lane striping, and the taper length was measured as the distance between the end of lane striping and the point where the lane width returns to that of a single lane. Both lengths were measured using Google Earth (Google 2021b) and rounded to the nearest 10 feet. The summed lengths (short lane length plus taper length) are given as the "Total Distance" value in Table 3-1. As shown in Figure 3-3, 18 of the 25 locations have a short lane 300 feet or less in length. The only location with a short lane longer than 1,000 feet was located at S.R. 36 and Saddleback Blvd.

Figure 3-2 Example of length measurements.

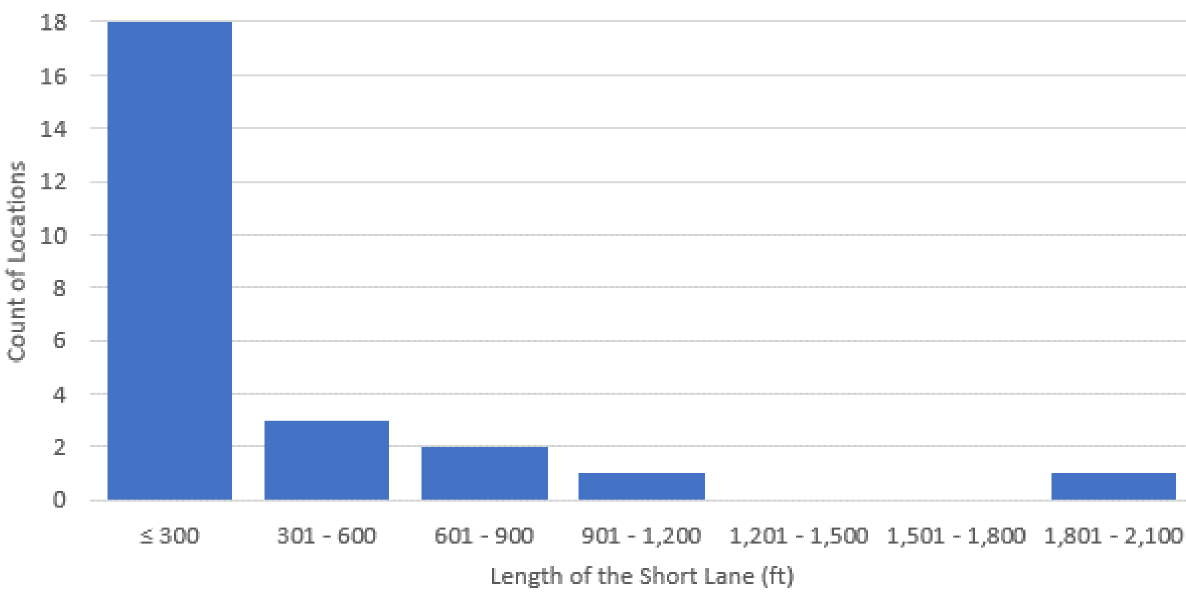


Figure 3-3 Distribution of short lane lengths.

3.4 Vehicle Data Collection

The vehicles approaching the intersection upstream of the lane drop locations were video recorded. All approaches were recorded from 4 PM to 6 PM on a mid-November weekday without inclement weather. The PM peak period (approximated as 4 PM to 6 PM for the purposes of this research) was used because it offered the highest traffic volumes of the day.

Where available, video recordings of the approaches were obtained from the UDOT Traffic Operations Center. In all other cases, video footage was recorded independently. All videos were cut at the hour mark, meaning that each approach had two videos—one for 4:00 PM to 5:00 PM and one for 5:00 PM to 6:00 PM.

While watching the video recordings, the data collection personnel would use one of three turning movement count apps to record each through vehicle that crossed the approach stop bar. Each of the apps recorded the timestamps and travel lane of every vehicle. If a shared through/right lane was present, the right-turning vehicles were also included, but they were marked as non-through vehicles.

The collected data were exported from each app for each video. The three apps used different codes to represent the lane number of each vehicle, and the key to these codes were noted. If the videos were watched and recorded at a higher speed, this was also noted. The timestamps were measured either as seconds from the start of counting or the time at which the vehicles were counted. None of the timestamps in the raw spreadsheets matched the time the vehicles physically passed the intersection since all the videos were counted after the fact. As will be explained in the following chapter, the artificial timestamps were converted to show the real timestamp of the vehicle.

3.5 Summary

A total of 25 study locations along the Wasatch Front were selected for study. The speed limit, number of lanes, and lengths of the short lane and taper were noted for each location. All locations had either two or three through lanes, and most of the locations had a speed limit of 40 mph and a short lane less than 300 feet long. Additionally, through vehicles approaching the signal in advance of the lane drop were recorded and counted.

4.0 DATA EVALUATION

4.1 Overview

This chapter discusses the analysis and evaluation of the collected data. Given first is an explanation of how the raw datasets were combined and then an explanation of how signal cycle breaks were identified in the data. Next is a description of the aggregations and calculations performed on the data. Following that is a discussion on the observational analysis which includes explanations of trends observed in the data. Finally, a discussion on the statistical analysis is given which includes numerical and graphical representations of the selected statistical model.

4.2 Combining the Raw Datasets

The first step in the data evaluation process was to combine the data into a single table. Due to the methods used in the data collection process, the vehicle data were saved in Microsoft Excel spreadsheets with each spreadsheet representing an hour of data collection. Each spreadsheet provided the name of the location, the hour counted (i.e., 4:00 PM to 5:00 PM or 5:00 PM to 6:00 PM), the timestamps of each vehicle, and the lane number code for each vehicle.

The timestamp of the first vehicle in each spreadsheet was converted to 4:00:00 PM or 5:00:00 PM, depending on the hour counted. The subsequent timestamps were converted to the real timestamps of the vehicles based on the gaps in the artificial timestamps. These gaps were calculated by subtracting the timestamp of the leading vehicle from the timestamp of the following vehicle. If a video had been watched at a higher speed, the gaps between the artificial timestamps were multiplied by the speed of the video before being used to calculate the real timestamps. The transformed data from the Excel spreadsheets were then entered into a Microsoft Access database.

Using Access, the datasets were combined into a single spreadsheet. Additionally, the lane number codes were converted into lane numbers using the keys noted during the data collection process. After these steps, the spreadsheet of vehicle data displayed the following for each vehicle: approach ID, timestamp, gap (in seconds) between that vehicle and the following vehicle, and lane number.

4.3 Identifying Cycles

The next step was to identify which vehicles belonged to the same cycle. Upon reviewing the data there were many approaches that had a clear size distinction between gaps during the green phases and the gaps indicating the red phases. A threshold was set for each approach so that gaps above the threshold indicated the beginning of a new green phase. For example, most of the gaps eastbound at 12th St and Wall Ave (Approach 3) were 15 seconds or less but approximately every two minutes there was a gap greater than 80 seconds. The threshold for this approach was set to 80 seconds, meaning that any vehicle following a gap of 80 or more seconds indicated the start of a new green phase. For approaches that did not have a clear distinction between gaps during the green phase and gaps indicating the red phase, the signal timing was reviewed on the Automated Traffic Signal Performance Measures (ATSPM) website, and the threshold was set to approximately the length of the red phase. Table 4-1 lists the thresholds set for each approach.

The first vehicle on each approach was assigned to cycle 1. Subsequent vehicles were assigned to the same cycle as the previous vehicle until a gap exceeded the threshold. A vehicle following such a gap was assigned to the next cycle. This process was continued until every vehicle on each approach had been assigned a cycle number.

In a separate table, the cycles on each approach were listed with their beginning and ending timestamps. The beginning timestamps were set equal to one second less than the timestamp of the first vehicle in that cycle. The ending timestamps were set equal to the beginning timestamp of the following cycle. A few unusually long (180 seconds or longer) cycles remained due to the imperfect process, but the number of these was minimized after referring to the ATSPM website as described.

Data for one approach (Approach 18, westbound at 9000 S and Redwood Rd) was collected differently than the rest. In the video recording for this approach, the stop bar was not visible; therefore, timestamps were recorded when the vehicles approached the intersection instead of when they crossed the stop bar. Because this meant that some timestamps represented an arrival on red, the gap sizes could not be used to determine the endpoints of the green phase. Instead, the timestamp of the beginning of the red phase (which was recorded for this approach during the data collection process) was used to mark the start of each new cycle for this approach.

Table 4-1 Minimum Gap Lengths Indicating a Cycle Break

ID	*Description*	*Direction*	*Minimum Gap Length Indicating a Cycle Break (sec)*
1	12th St & Wall Ave	NB	45
2	12th St & Wall Ave	SB	55
3	12th St & Wall Ave	EB	80
4	12th St & Wall Ave	WB	75
5	5600 S & 3500 W	NB	35
6	5600 S & 3500 W	SB	40
7	5600 S & 3500 W	WB	60
8	U.S. 89 & Skyline Dr	SB	55
9	S.R. 39 & S.R. 126	NB	55
10	S.R. 36 & Saddleback Blvd	SB	20
11	1000 N & Redwood Rd	NB	21
12	1300 S & State St	EB	70
13	2300 E & Foothill Dr	EB	40
14	2100 S & 1300 E	NB	50
15	S.R. 111 & 7800 S	NB	21
16	8000 S & State St	SB	25
17	9000 S & Redwood Rd	EB	85
18	9000 S & Redwood Rd	WB	*
19	9000 S & 700 W	WB	26
20	9000 S & State St	NB	100
21	S.R. 92 & 4800 W	EB	55
22	AF Main St & State St	EB	40
23	1600 N & State St	WB	95
24	Center St & 1200 W	EB	36
25	University Pkwy & Geneva Rd	SB	40

* The start of each cycle on Approach 18 was determined from timestamps of the cycle breaks collected for that approach during the data collection process.

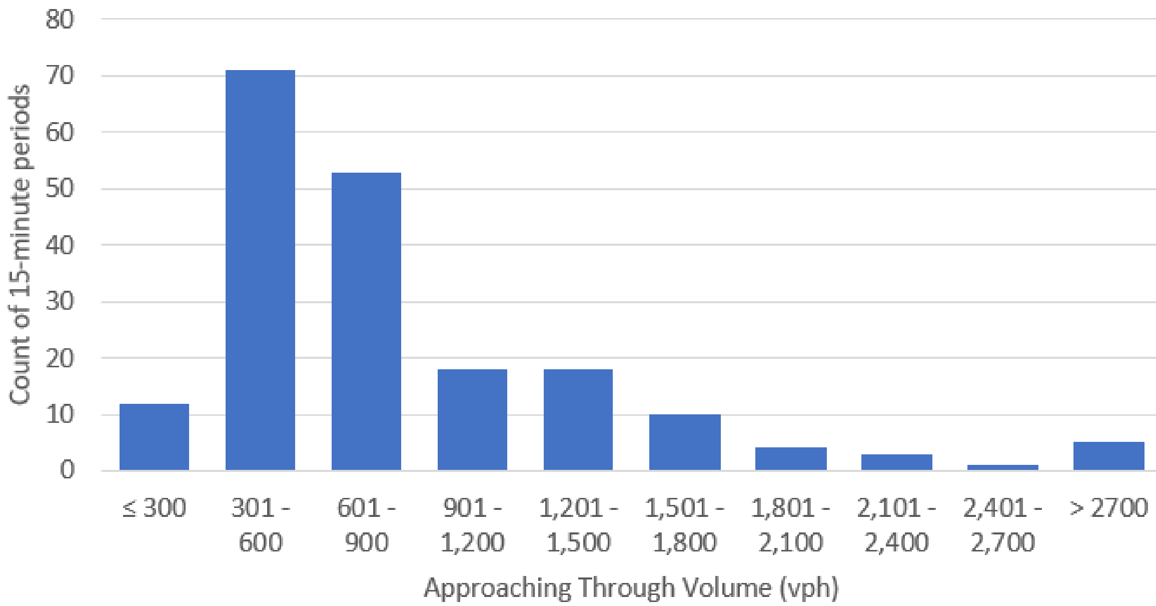
4.4 Aggregations and Calculations

After vehicles were assigned to cycles, the number of vehicles in each lane were then summed up for each cycle, and the average volume per lane was calculated by dividing the total approach volume by the number of approach lanes. The utilization rate of the short lane was then calculated for each cycle by dividing the short lane volume by the average volume per lane as shown in Equation 4-1.

$$Utilization\ Rate = \frac{Short\ Lane\ Volume}{Average\ Volume\ per\ Lane} \tag{4-1}$$

The utilization rate is equal to 1 when the short lane is used by one half of the approaching vehicles for two-lane approaches and by one third of the approaching vehicles for three-lane approaches. A utilization greater than 1 indicates that the short lane is favored over the continuous through lanes, and a utilization less than 1 indicates that the continuous through lanes are favored over the short lane.

The number of vehicles were then summed up by 15-minute periods. Vehicles in the same cycle were kept together—the vehicles in a cycle were assigned to the 15-minute period in which the cycle began. The average volume per lane and utilization rate were then calculated for the 15-minute aggregation. As shown in Figure 4-1, most approaches had volumes between 300 and 900 vehicles per hour.



Note: 15-minute volumes were multiplied by four and are displayed as hourly flow rates.

Figure 4-1 Distribution of hourly approach volumes.

4.5 Observational Analysis

Trends in the data were analyzed to better understand which variables might be important to test in a statistical model. This section looks at both the cycle and 15-minute aggregations.

4.5.1 Cycle-Level Observations

For the cycle-level observations, the data was refined two steps further. First, the first and last cycle on each approach were removed since they did not necessarily represent an entire green-to-red period. Second, any cycles that lasted 180 seconds or longer were removed because they likely represented two cycles as one, thus incorrectly showing an unusually high volume for the observed utilization rate.

The first relationship explored was that of the utilization rate versus the total approach volume. As shown in Figure 4-2, some cycles had a utilization rate greater than 1 when the total approach volumes were 11 vehicles or less. For cycles with total approach volumes below 20 vehicles, the utilization rate appears to decrease as the total approach volume increases, while the same trend does not appear to be the case for cycles with more than 20 vehicles. After the volume in the cycle surpasses 20 vehicles, it appears that the utilization rate increases as the total approach volume increases. The distinct grouped curves in the chart come from the volume of the short lane: All points in the bottom curve had one vehicle in the short lane; all points in the second-to-bottom curve had two vehicles in the short lane, and so on.

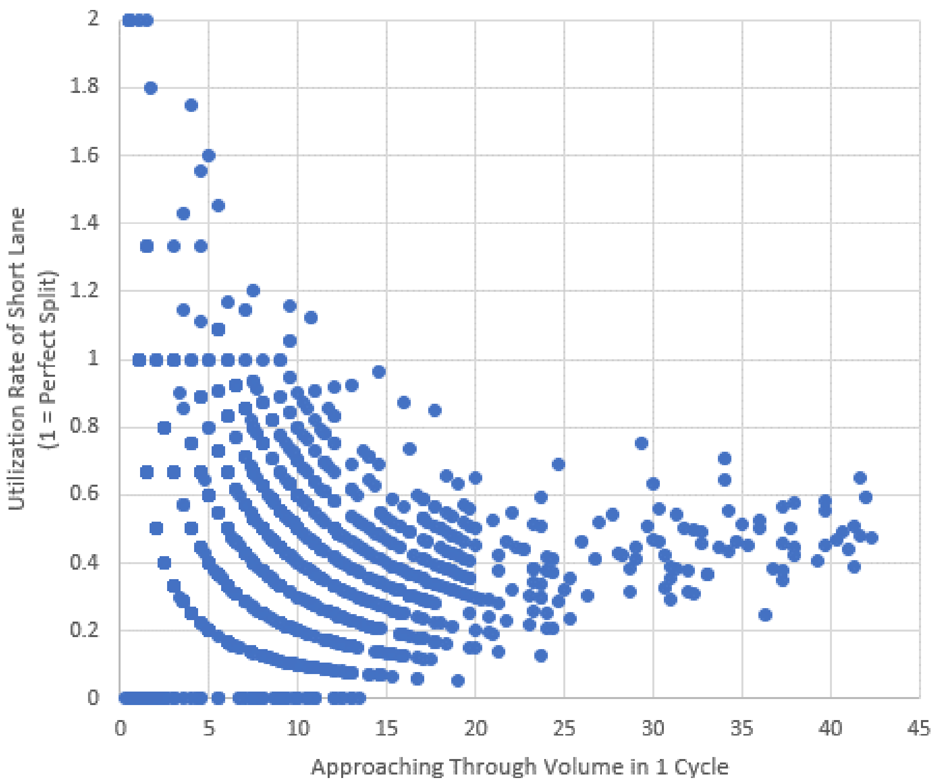
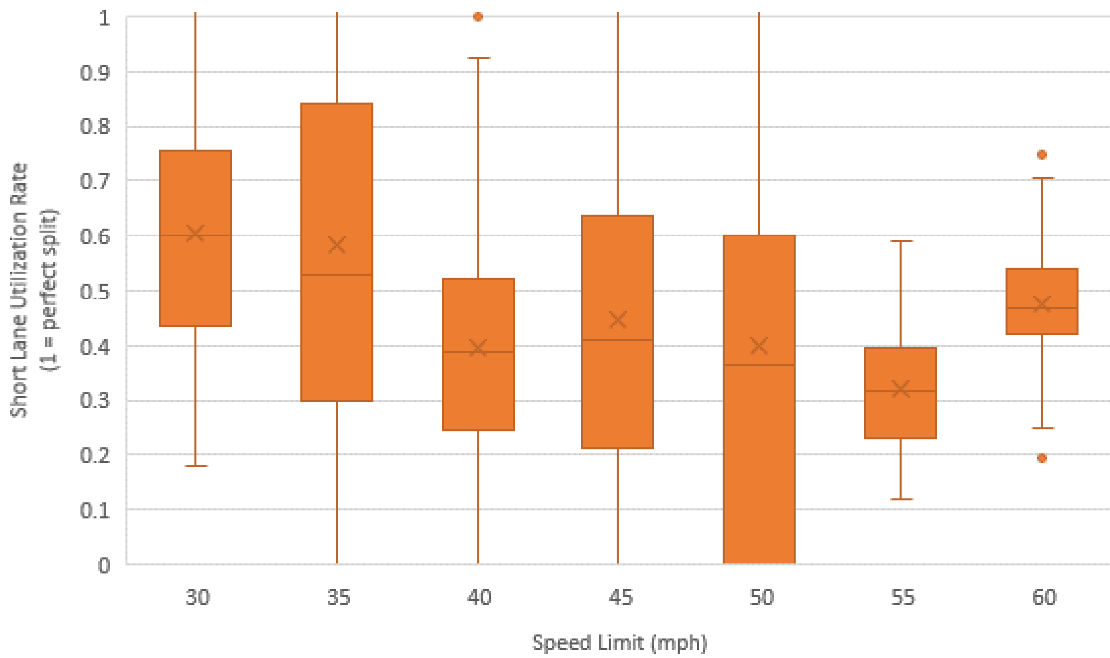


Figure 4-2 Utilization rate versus approach volume.

The second relationship explored was that of the utilization rate versus speed limit. As shown in Figure 4-3, the utilization rate generally decreases as the speed limit increases. This may indicate that drivers are more comfortable merging at lower speeds than at higher speeds.



Note: For this and all box and whisker plots in this report, the middle 50% of the data falls within the colored rectangles, the horizontal bar in the rectangle represents the median, the X represents the mean, and the round points represent any outliers.

Figure 4-3 Utilization rate versus speed limit.

The third relationship explored was that of the utilization rate versus length of the short lane. As shown in Figure 4-4, the utilization rate generally increases as the short lane increases in length.

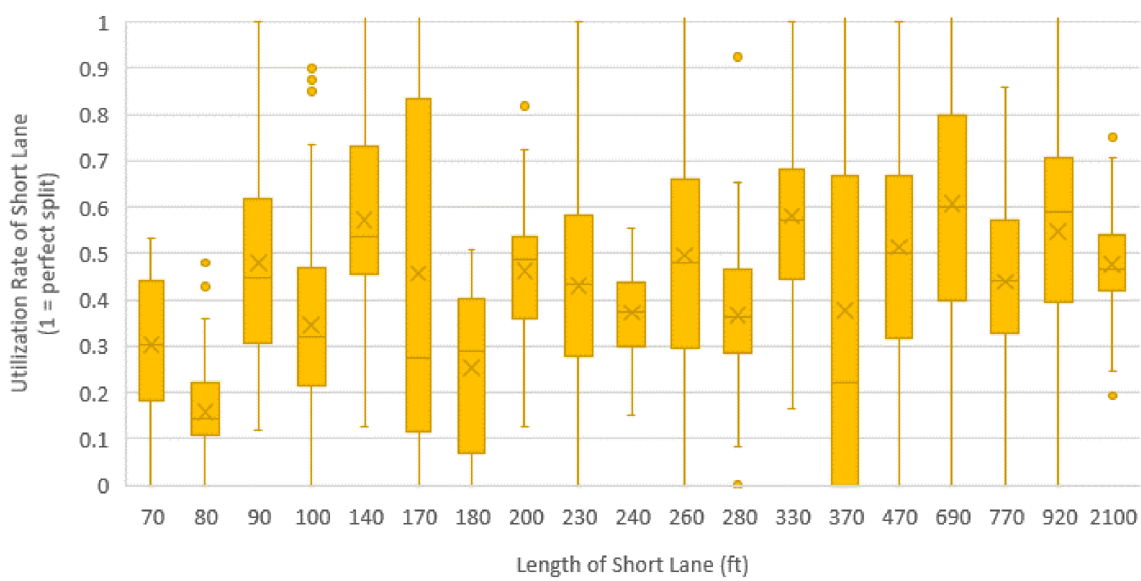


Figure 4-4 Utilization rate versus striped length.

The fourth relationship explored was that of the utilization rate versus special lane drop type. The three special lane drop types analyzed were trap lane, through/right lane, and auxiliary lane. As can be seen in Figure 4-5, the utilization rate appears to be lower for trap lanes and through/right lanes.

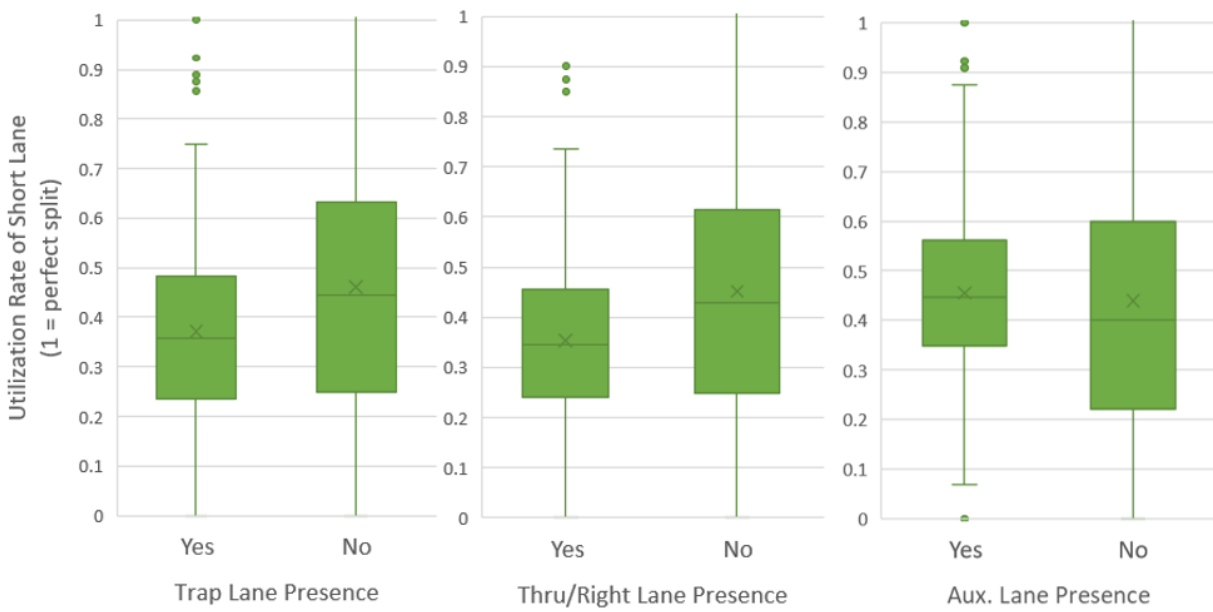


Figure 4-5 Utilization rates for different special lane drop types.

4.5.2 15-Minute Period Observations

For observations discussed in this section, the vehicles were aggregated by 15-minute periods as explained previously in section 4.4. The data refinement for the cycle-level observations as explained in section 4.5.1 was not needed for the aggregation by 15-minute periods and was therefore not performed for the observations in this section.

The first relationship explored with the 15-minute periods was that of the utilization rate of the short lane versus the total approach volume. For this relationship, the volume of each 15-minute period was multiplied by four to get an hourly flow rate. As shown in Figure 4-6, there were no 15-minute periods that had a utilization rate greater than 1. Periods with total approach volumes below 750 vehicles per hour appear to have lower utilization rates as the total approach volume increases. After the hourly volume surpasses 1,500, it appears that the utilization rate increases as the total approach volume increases. Because of these two trends, a curve fits the data much better than a straight line as evidenced by the equations displayed in the figure. This indicates that the volume may need a transformation when added to a statistical model.

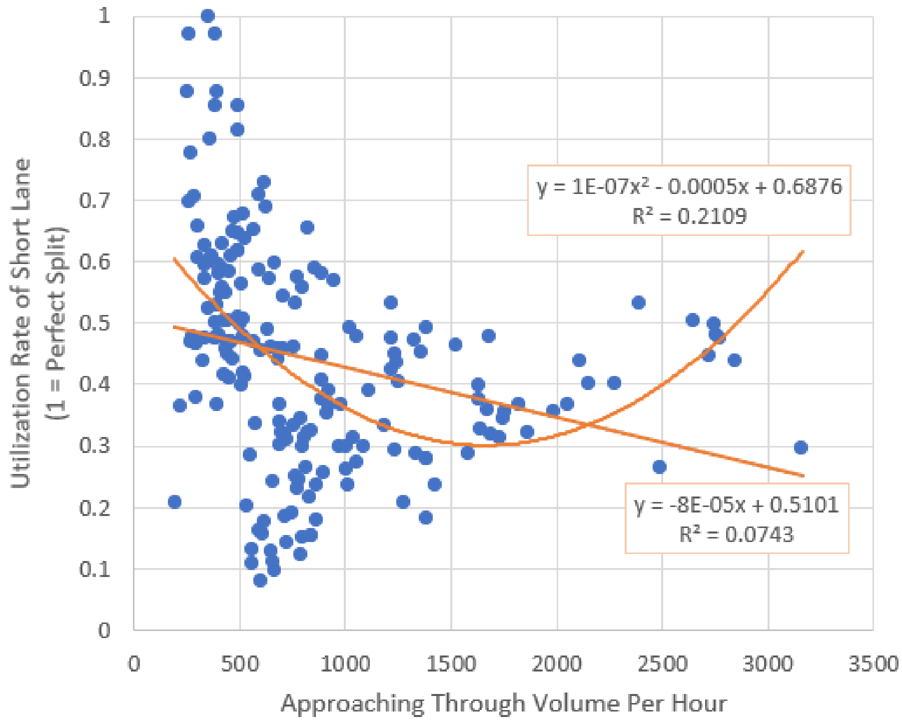


Figure 4-6 Utilization rate versus approaching volume.

The second relationship explored was that of the utilization rate versus speed limit. As shown in Figure 4-7, the utilization rate generally decreases as the speed limit increases. As stated in the cycle-level observations, this trend may indicate that drivers are more comfortable merging at lower speeds than at higher speeds.

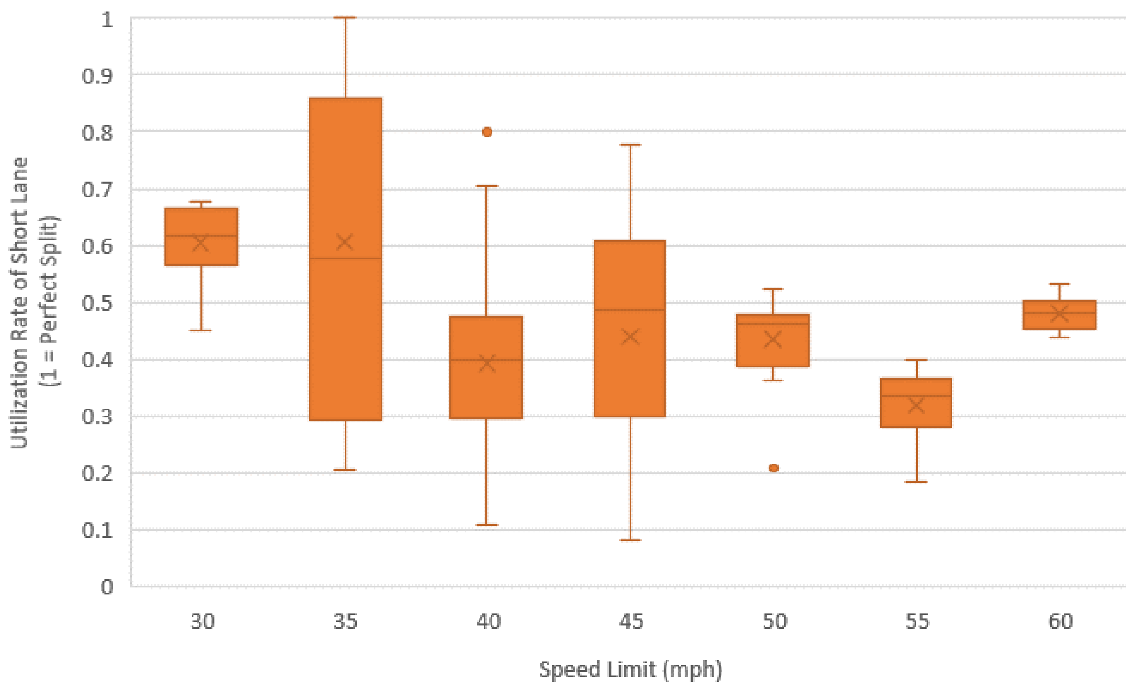


Figure 4-7 Utilization rate versus speed limit.

The third relationship explored was that of the utilization rate versus length of the short lane. As shown in Figure 4-8, the utilization rate generally increases as the striped length increases, similar to the trend found in the cycle-level analysis.

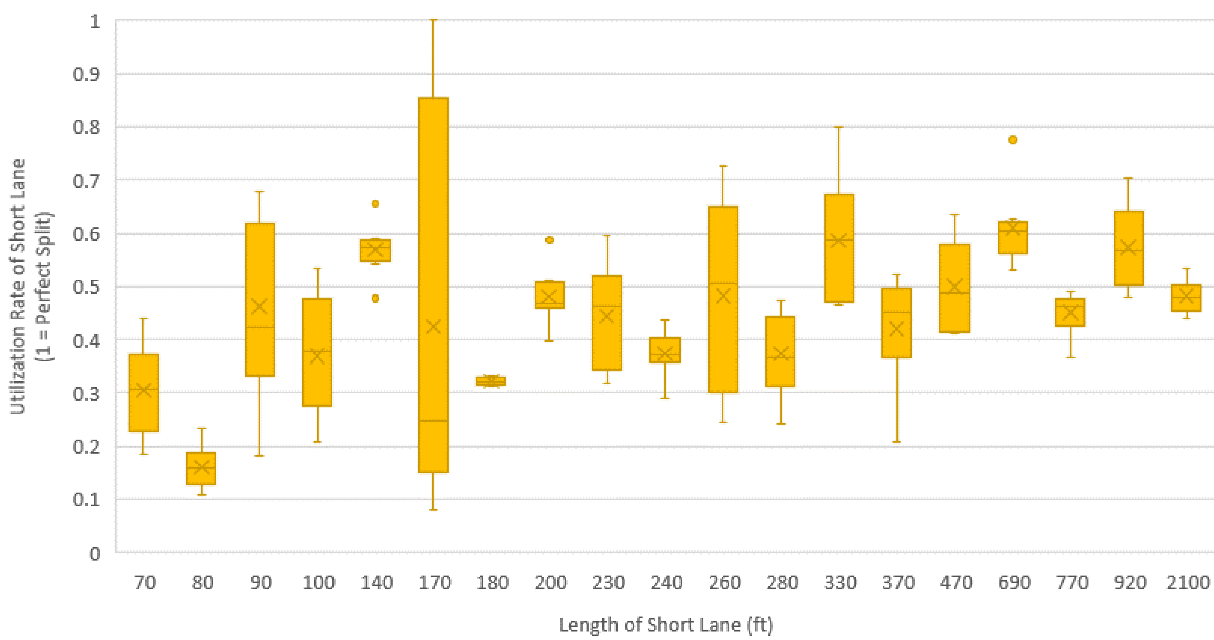


Figure 4-8 Utilization rate versus the length of the short lane.

The fourth relationship explored was that of the utilization rate versus special lane drop type. Just like in the cycle-level observations, the three special lane drop types analyzed were trap lane, through/right lane, and auxiliary lane. As can be seen in Figure 4-9, the utilization rate appears to be lower for trap lanes and through/right lanes.

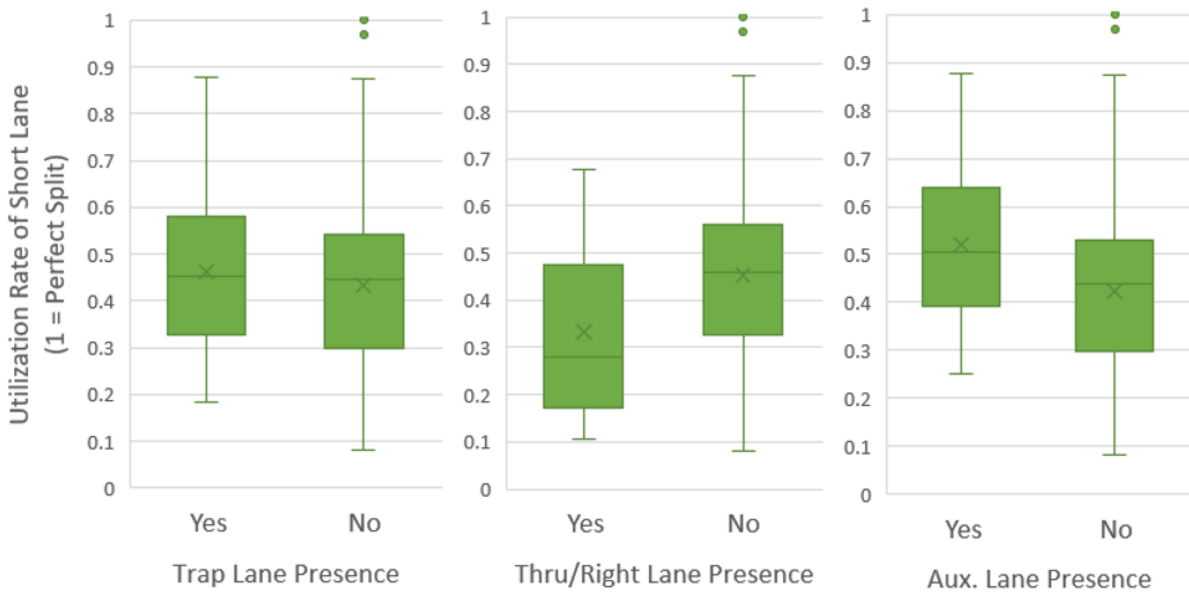


Figure 4-9 Utilization rates for different special lane drop types.

4.6 Statistical Analysis

A statistical analysis was performed on the data to build upon the findings from the observational analysis and to quantify the impacts of roadway and volume characteristics on the utilization rate. The benefits of building statistical regression models are that several variables can be assessed for their simultaneous impacts on lane utilization and the regression models can be used for predictive purposes. Statistical regression models were estimated using RStudio and R version 4.0.2.

4.6.1 Statistical Methods

Ordinary Least Squares models were preferred over other more advanced models since the observed lane utilization rates were widely dispersed between 0 and 1 and the immediate usability for practitioners would be much greater than with more advanced models such as logistic regression. Candidate models were estimated including variables identified from the observational analysis, including approaching through volume, number of lanes, short lane length, taper length, speed limit, and the presence of a trap lane, auxiliary lane, and/or a combined through/right lane. It was observed that the approach at S.R. 36 & Saddleback Blvd was the only approach with a speed limit of 60 mph and had the longest short lane by more than double (2,100 feet vs. 920 feet). For these reasons, this approach was excluded from these models as an outlier; without this action, the statistical model would not reliably decipher between the impact of the two variables with only one cluster of data points.

Data transformations were investigated for volume and short lane length. As mentioned previously, a non-linear trend is observable for the impact of volume on the utilization rate. After investigation, it was determined that models using a squared transformation of volume or a natural log transformation of volume resulted in similar model fits. However, the conceptual interpretation of covariates of interest was preferable when volume was log-transformed, so the natural log transformation was preferred over the squared transformation. The major difference between a natural log transformation and a squared transformation is at the more extreme values, where at higher volume values the squared curve would flatten out and turn up again, while the natural log curve would flatten out and approach a slope of zero. The lack of high-volume data points is a major hinderance in the ability of the regression model to properly capture the curvature of the trend. It is possible that if the models were estimated again with more data points with high volumes that a more defined curvature would lead to a preference for the squared transformation.

Stepwise regression models were estimated to determine the specifications of the best-fit models. Several of these model specifications investigated are presented in Table 4-2. Cluster-robust standard errors were used for determination of statistical significance to account for repeated observations (15-minute intervals) of the same approaches. Models including either vehicles per hour or vehicles per hour per lane were estimated, though only the vehicles per hour measure was significant in any models, and only borderline significant in the final model. The number of lanes was also not significant in any of the models. The length of the short lane, the presence of a trap lane, and the speed limit were significant in most or all estimated models. It should be noted that the taper distance was also included for some models, but it was highly correlated with speed limits, as taper lengths are generally designed based on the speed limit or design speed. Speed limit typically explained slightly more variation than taper length, therefore it was preferred over taper length.

Table 4-2 Stepwise Regression Models with Variables Tested

Variable	*Model 1*	*Model 2*	*Model 3*	*Model 4*	*Model 5*	*Model 6*	*Model 7*	*Final*
$ln(vehicles\ per\ hour)$	*	NS	NS					#
$ln(vehicles\ per\ hour\ per\ lane)$				NS	NS			
Number of Lanes			NS			NS		
Short Lane Length (in ft)	NS	*	NS	***	**	NS	***	*
Trap Lane		***	***	***	**	**	***	**
Speed Limit	#	**	**	**	*	*	*	*
Auxiliary Lane		NS	NS	NS			NS	
Combined Through/Right Lane		NS	NS	NS				

Note: NS = Not Significant, # = Significant at $p \leq 0.1$, * = Significant at $p \leq 0.05$, ** = Significant at $p \leq 0.01$, *** = Significant at $p \leq 0.001$

4.6.2 Selected Model

The final regression model estimates are presented in Table 4-3. The length of the short lane was positively associated with lane utilization, indicating that longer approaches to lane drop locations are more conducive to higher upstream intersection lane utilization rates, with each extra 100 feet before the lane drop location corresponding to a 0.024 increase in the utilization rate. Volume, the presence of a trap lane, and speed limit were negatively associated with lane utilization, with each unit increase in the natural log of vehicles per hour being associated with a 0.103 drop in the utilization rate (corresponding to a 0.071 drop for doubling the volume), the presence of a trap lane corresponding to a 0.108 drop in the utilization rate, and a 0.0125 drop in the utilization rate being associated with each additional mph in speed limit (corresponding to a 0.0625 drop for an increase in 5 mph).

Table 4-3 Regression Coefficient Estimates for Lane Utilization

Variable	*Estimate*	*Std. Error*[1]	*T-value*	*P-value*
Intercept	1.59	0.508	3.13	0.002
ln(vehicles per hour)	-0.103	0.0566	-1.83	0.067
Short Lane Length (in 100 ft)	0.0243	0.0118	2.06	0.039
Trap Lane	-0.108	0.0375	-2.87	0.004
Speed Limit	-0.0125	0.0057	-2.18	0.029
R^2	0.374			
Adj R^2	0.360			
N	189			

[1]Cluster-Robust Standard Error

The resulting regression equation is given in Equation 4-2:

$$U = 1.59 - 0.103 \times \ln(vph) + 0.0243 \times L - 0.108 \times T - 0.0125 \times S \quad (4\text{-}2)$$

Where U = the lane utilization rate,

$vph =$ the approach volume in vehicles per hour,

L = the length of the short lane in 100s of feet,

T = logical variable for trap lane, 1= trap lane present, 0= no trap lane present, and

S = speed limit in miles per hour.

Practitioners can use this equation as an estimated lane utilization when designing (or redesigning) intersections with lane drops. It must be noted, however, that these estimates are based on the average lane utilizations observed in this study. Despite the significant predictive power of the model, there was still much variation between the model predictions and observed utilization values. Practitioners who use this estimation equation should use engineering judgment to account for qualitative or other factors not addressed by this equation. Caution should also be exercised regarding extrapolating this equation or using it in scenarios outside the realm of the data that contributed to its estimation, such as for longer short lanes, higher volumes, or speed limits that were not represented in this sample.

The actual lane utilizations observed by the length of the short lane and volume bins are presented in Figure 4-10. The model predicted lane utilization by the length of the short lane and volume bins are presented in Figure 4-11. The predicted utilization rates follow the same trend as the actual lane utilization rates, though much of the variation is muted. It is apparent from these figures that there are only a handful of observations with longer short lanes (>600 feet) as well as only a handful of high-volume observations (1600+ vph). The applicability of the model would be expanded with the addition of more data points in these two areas, particularly points that are in both.

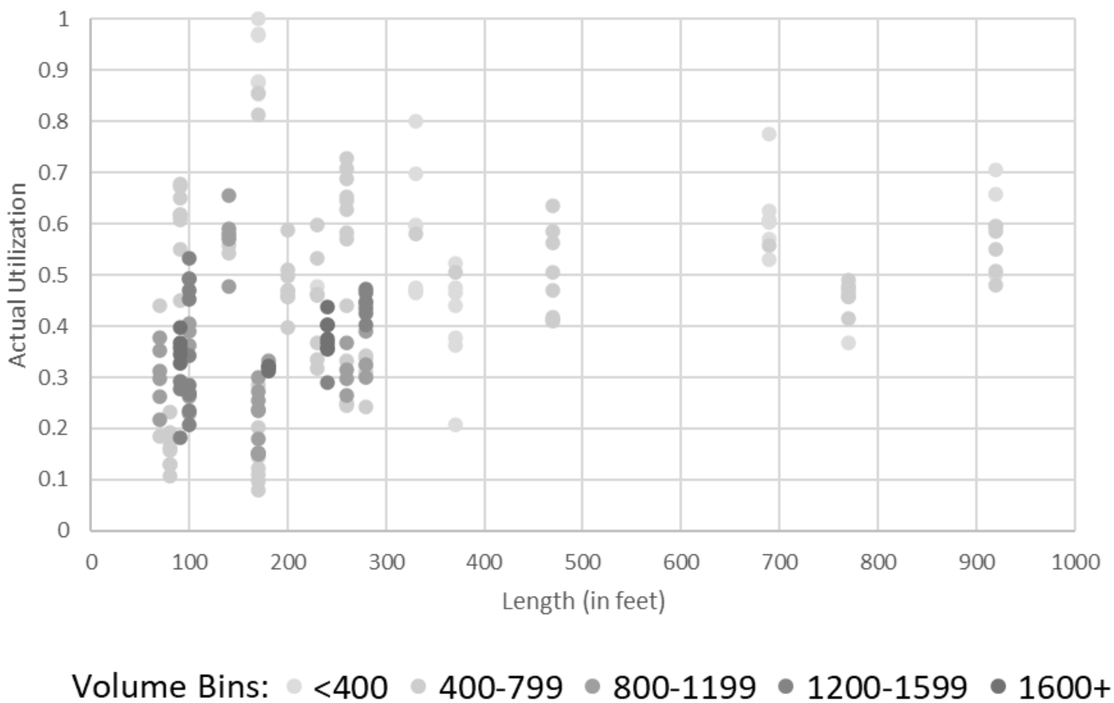


Figure 4-10 Actual lane utilization rates by length of the short lane and volume.

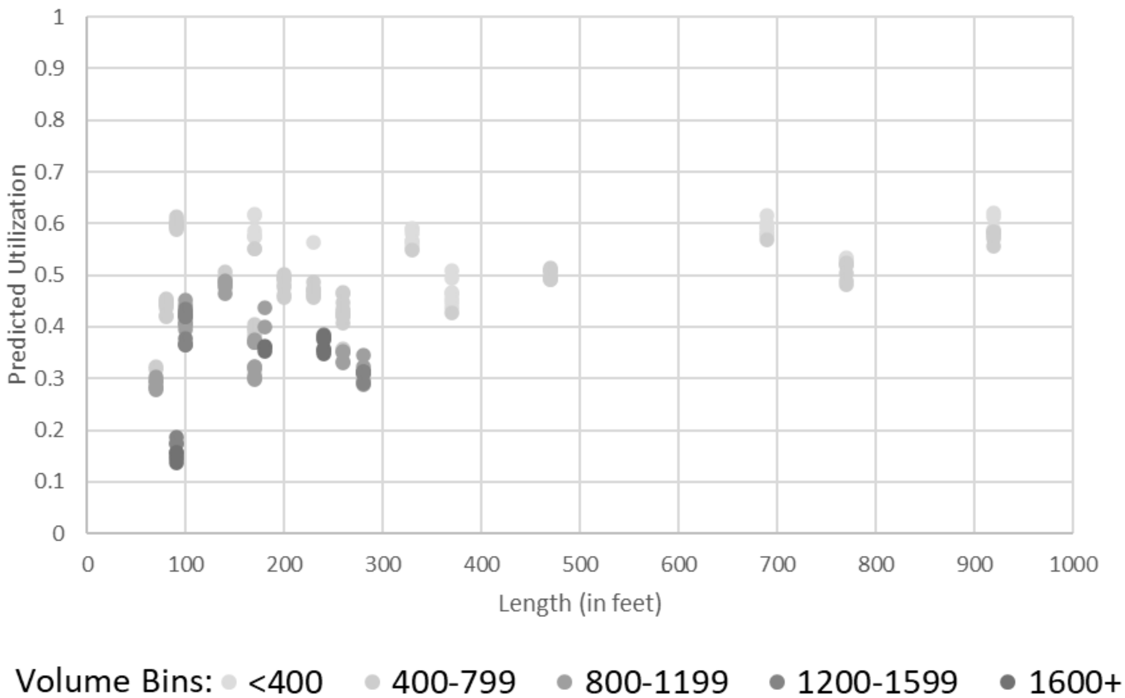


Figure 4-11 Predicted lane utilization rates by length of the short lane and volume.

Figure 4-12 and Figure 4-13 are graphical representations of the utilization rate equation based on the length of the short lane and hourly through volume superimposed over the observed values. These charts assume the roadway has a speed limit of 40 mph and that the study location is not a trap lane. If these assumptions are not met, the utilization rate should be adjusted as follows: Subtract 0.0625 for each 5 mph increase in the speed limit (or add 0.0625 for each 5 mph decrease in the speed limit), and/or subtract 0.108 for a trap lane.

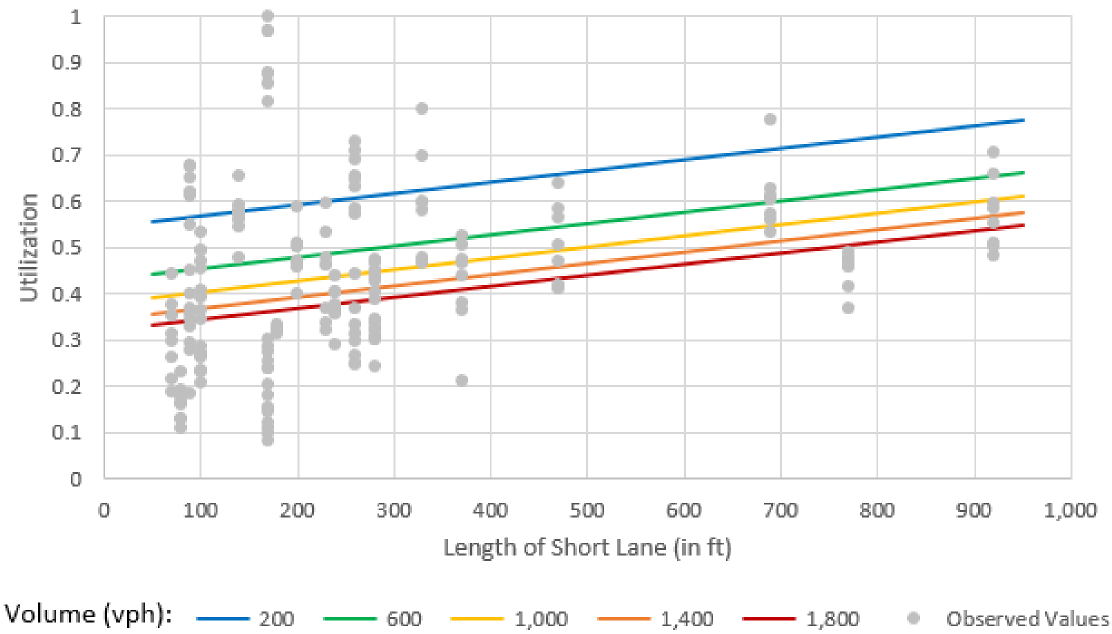


Figure 4-12 Predicted utilization based on the short lane length by hourly volume.

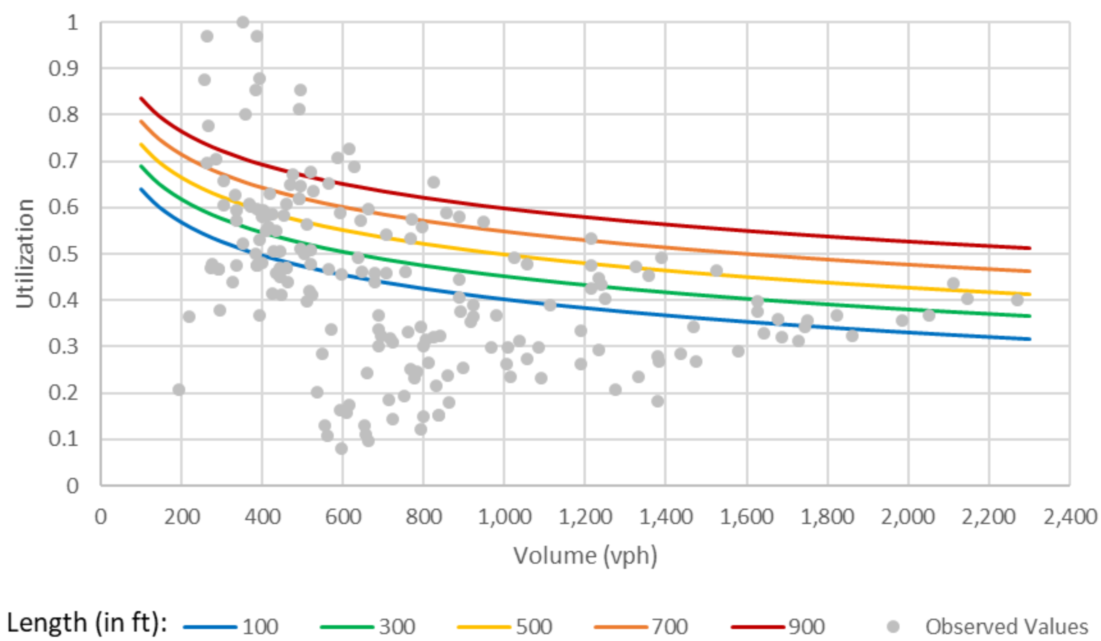


Figure 4-13 Predicted utilization based on hourly volume by the short lane length.

The observed values give the user a sense of the amount of data that supports a calculation. For example, Figure 4-12 shows that short lanes between 200 and 300 feet long are well represented while short lanes 600 feet long are largely unrepresented. Likewise, Figure 4-13 shows that approaches with 600-800 vph are well-represented while approaches with 2,300 vph are unrepresented. The majority of ranges that are well represented were locations that observed relatively low lane utilizations; this trend is reflected in the model which predicts lane utilizations lower than 0.6 for most short lanes.

With proper judgment, practitioners can use Figure 4-12 and Figure 4-13 to estimate lane utilizations based on the length of the short lane and volume, potentially even estimating how much longer a lane drop would need to be to make lane utilization more acceptable, though the ability of these figures is limited with the skewedness of the data toward shorter short lanes. A specification for a minimum short lane length that can produce acceptable lane utilization cannot be reasonably applied to these findings as most of the observations had lane utilization rates less than 0.6, indicating that if there is a preferable standard for length of the short lane it is likely beyond the lengths represented by most of the sample.

4.7 Summary

After some initial data processing, an observational analysis was performed. In this analysis, the variables of speed limit, length of the short lane, approaching through volume, and lane drop type were identified as ones that showed a potential for predicting lane utilization. Based on the observational analysis, a statistical analysis was performed which fit a regression model to the data. This chapter presented the selected regression model as well as graphical representations of the model for use in accordance with engineering judgment. The following chapter will present the conclusions of the research.

5.0 CONCLUSIONS

5.1 Summary

The primary objective of this research is to determine the effects that the distance from a signalized intersection to a lane drop location has on lane utilization. Lane-by-lane counts were collected from 4 PM – 6 PM for 25 approaches located along the Wasatch Front. These data were then used to calculate the utilization rate of the short lane for each cycle and for each 15-minute period. Both observational and statistical analyses were performed to assess the relationship between lane utilization and a number of other factors, including short lane length, volume, speed limit, and lane drop type. An ordinary least squares statistical model was then fit to the data using a stepwise process.

5.2 Findings

The final regression model predicted lane utilization based on four variables: short lane length (positive coefficient), natural log of hourly through volume (negative coefficient), presence of a trap lane (negative coefficient), and speed limit (negative coefficient). These findings suggest that higher volumes, the presence of a trap lane, and higher speed limits decrease lane utilization upstream of the intersection, while longer short lanes tend to increase lane utilization. The model equation and its graphical representations were given in section 4.6.2.

Most of the observations to which the model was fitted had short lanes of relatively short lengths (300 feet or shorter) with relatively small approach volumes (900 vph or less). Moreover, most of the observations were associated with lane utilization rates less than 0.6. These findings indicate that the predictive power of the model is skewed towards undesirable lane drop scenarios and that if there is a preferable short lane length for acceptable lane utilizations it likely exceeds the lengths represented in the sample.

5.3 Limitations and Challenges

The limitations of the model came as a result of the sample size and skewedness of the short lane lengths and approaching through volumes for locations selected in this study. Half of the selected locations have the same speed limit of 40 mph, three-quarters have short lanes of 300 feet or less, and more than two-thirds have through volumes less than 900 vph. This limitation was not foreseen, and the location selection process was mostly driven by cost-effectiveness of data collection. While the sample of approaches included in this study may be representative of lane drop locations in Utah, they are skewed towards shorter lengths and approaches with relatively low volumes. This skewedness limits the reliability of the interpretations from these models to situations outside of the context of the data. Therefore, caution should be exercised when implementing the findings of these models for approaches with over 1,000 through vehicles per hour or for short lanes longer than 400 feet. Unfortunately, the need for analysis of lane utilization is typically greatest for approaches with high volumes. Additionally, the short lanes observed did not generally appear to be long enough to produce desirable lane utilizations. Thus, the model estimated in this study may be made more useful after addressing the lack of data representing medium to long short lanes and high-volume approaches.

6.0 RECOMMENDATIONS

6.1 Recommendations

It is recommended that the selected regression model, presented in Equation 4-2, be used to predict lane utilization for lane drop locations in Utah with short lanes less than 400 feet long and on approaches with less than 1,000 through vehicles per hour. Within these bounds, the model indicates that increased lane utilizations are predicted for lane drop locations with longer distances from the signalized intersection to the end of the short lane, lower speed limits, lower volumes, and the absence of a trap lane. Engineering judgment should be applied to the use of the model, and an understanding of the limitations of the model should be had before attempting to extrapolate results. Unfortunately, lane utilization rates less than 0.6 were observed in most of the approaches studied, indicating that if there is a minimum desirable short lane length, it is likely longer than what is represented in the sample.

6.2 Further Research Opportunities

Further understanding of lane utilization associated with lane drops could be obtained through additional research efforts. Three such opportunities are expanding the study to include more lane drop locations, exploring new factors that may influence lane utilization, and correlating lane change distances with resultant lane utilization in simulation software.

6.2.1 Including Additional Lane Drop Locations

One opportunity for further research on this topic is to improve the model by increasing the sample size to include a well-rounded selection of lane drop locations. Ideal candidates to add to the current sample would have short lanes longer than 300 feet (preferably including many that are longer than 900 feet) and/or lane drop approaches with through volumes greater than 900 vph.

There are no known reference files from UDOT or local Utah municipalities that list lane drop locations. In order to identify additional lane drop locations, less-efficient methods must be employed. These methods include referring to personal experience/memory and using a pavement-messages or sign-faces file to identify merging locations. With an initial attempt at these methods, the research team has identified 16 additional locations in Utah that meet at least one of these criteria. These locations are listed in Table 6-1.

Table 6-1 Additional Identified Locations

UDOT Region	*County*	*City*	*Location*	*Coordinates*	*Striped Length (ft)*
1	Weber	West Haven	S.R. 108 @ Hinckley Dr	41.1981, -112.039	320
1	Cache	Logan	S.R. 252 (1000 W) @ 1400 N	41.7586, -111.859	360
2	Salt Lake	Magna	S.R. 171 (3500 S) @ S.R. 111	40.6964, -112.091	410
2	Salt Lake	Murray	S.R. 71 (700 E) @ S.R. 266 (4500 S)	40.6737, -111.872	830
2	Salt Lake	Sandy	S.R. 209 (Little Cottonwood Rd) @ Eastdell Dr	40.5782, -111.823	540
2	Salt Lake	Near Magna	S.R. 201 before S.R. 202	40.7277, -112.169	2,200
2	Summit	Park City	S.R. 248 (Kearns Blvd) & Round Valley Dr	40.6637, -111.500	1,200
2	Summit	Park City	S.R. 248 (Kearns Blvd) & Monitor Dr	40.674, -111.470	360
2	Salt Lake	Salt Lake City	West Temple @ 900 S	40.748, -111.895	490
2	Salt Lake	Salt Lake City	4000 W @ 4100 S	40.681, -111.987	320
2	Salt Lake	Draper	S.R. 71 (12300 S) @ Lone Peak Pkwy	40.527, -111.897	640
3	Wasatch	Heber	U.S. 40 @ U.S. 189	40.4925, -111.413	330
3	Uintah	Naples	U.S. 40 @ S.R. 45	40.4141, -109.498	800
3	Utah	Mapleton	U.S. 89 @ 1600 N	40.144, -111.602	380
3	Utah	Provo	900 E @ Temple View Dr	40.262, -111.645	850
4	Millard	Delta	U.S. 6 (Main St) @ 400 W St	39.3526, -112.583	1,160

Figure 6-1 compares the current distribution of short lane lengths to the potential distribution if these 16 locations were added. Though still not an ideal distribution, the addition of these locations would increase the predictive power of the model for lane drop locations with short lanes longer than 300 feet. Obtaining a more complete list of lane drop locations in Utah would likely provide even greater benefit to the model. Moreover, signalized approaches to freeway ramps could also be included in the sample and could provide many lane drop locations with short lanes longer than 300 feet.

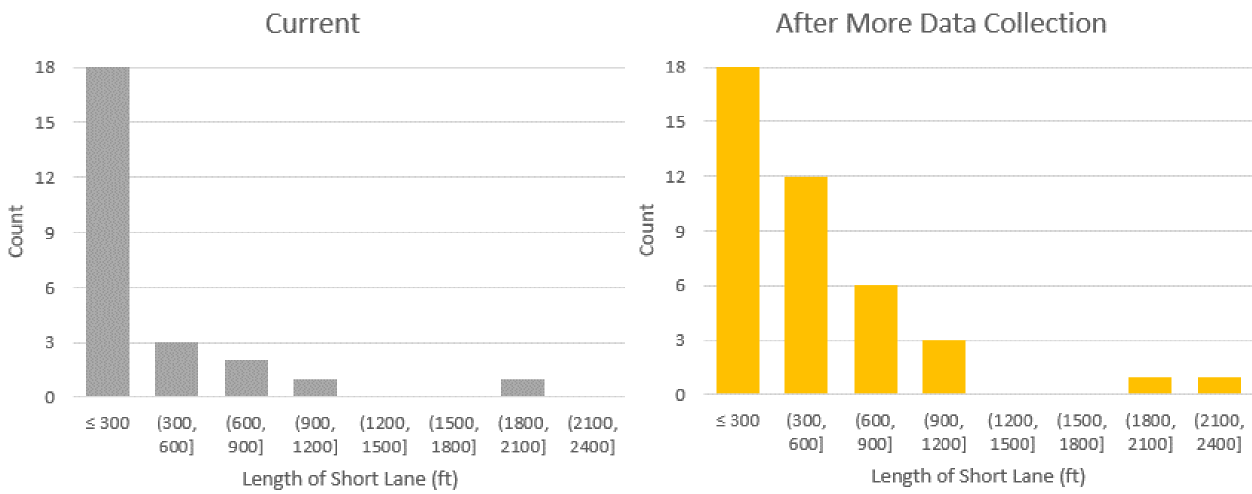


Figure 6-1 Comparing current sample size to potential sample size.

6.2.2 Exploring New Factors

The literature review identified more variables than were analyzed in this study. Some of the variables that may be useful to consider in a separate study include heavy vehicle percentage, land use, and ratio of green time to cycle length. Further research could also analyze other factors, such as signage, that may help mitigate low lane utilization associated with lane drops.

6.2.3 Correlating Lane Change Distance Settings

Some traffic simulation software, including VISSIM, provide a "lane change distance" setting that engineers may adjust when calibrating their models to match observed conditions. Further research efforts could be devoted to creating tables that recommend lane change distances based on anticipated lane utilization. These tables would be useful for engineers modeling projects that include lane drop locations near signalized intersections.

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